APPENDIX B

GEOTECHNICAL FEASIBILITY STUDY

PREPARED BY:
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JANUARY 2006

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Geotechnical Feasibility Consultation

East Sunnyvale ITR Project
Sunnyvale, California

Report No. 858-47A has been prepared for:
David J. Powers & Associates
San Jose, California

January 31, 2006

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Geotechnical Feasibility Consultation

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GEOTECHNICAL FEASIBILITY CONSULTATION
EAST SUNNYVALE ITR PROJECT
SUNNYVALE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical feasibility consultation for the future development of the East Sunnyvale ITR Project to be located in Sunnyvale, California. The location of the site, along with sites where we previously performed geotechnical investigations, is shown on the Vicinity Map, Figure 1.

The purpose of our consultation was to review available published data and information from our previous investigations at portions of the site and in the site vicinity to provide information regarding the anticipated geotechnical conditions and potential geologic hazards that might impact the site development, and to provide a discussion of anticipated geotechnical concerns and different foundation options.

We are concurrently performing an Environmental Hazardous Material Evaluation for the site. The results of our environmental evaluation will be presented under a separate cover.

1.1 Previous On-Site Work

We previously performed several geotechnical investigations for portions of the project site and within the site vicinity, and presented the findings in our geotechnical reports. A reference list is provided at the end of this report. The approximate locations of our previous field investigations are shown on the Vicinity Map, Figure 1. We also provided geotechnical observation and testing services during construction of some of the projects. Results of our previous investigations and observations during construction were used to develop some of the preliminary recommendations presented in this report.

1.2 Project Description

The approximately 130-acre site contains industrial and office buildings with associated parking lots and landscaped areas. The area is in close proximity to three Superfund sites, and active remediation of ground water contamination is reportedly occurring. We understand that the City of Sunnyvale is evaluating a General Plan Amendment for the area to change the designation from Industrial with an M-S (Industrial and Service) designation to a designation of Industrial to Residential (ITR) that would allow industrial, office, commercial and residential uses. The area would then gradually transition to residential developments.

1.3 Scope of Services

Our scope of services was presented in detail in our agreement with you dated August 15, 2005. To accomplish this work, we provided the following services:

- Review of available published data and information in our files from previous investigations at the site and in the site vicinity.
- Preliminary engineering analysis to evaluate potential geotechnical and geologic hazards and evaluate preliminary foundation options.
- Preparation of this report to summarize our findings and to present our preliminary conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Summary of Site Experience

As discussed above, we have performed several geotechnical investigations within the project site and in the site vicinity. Figure 1 shows the approximate location of the site and the approximate locations of our previous geotechnical investigations within approximately ½-mile of the site. To evaluate the anticipated subsurface conditions at the site and potential geologic hazards, we reviewed the information in our files for the nine nearest and most applicable projects.

2.2 Surface

We also performed a brief surface reconnaissance. The site is located generally bounded by Duane Avenue on the north, Stewart Drive on the south, Wolfe Road on the west and Lawrence Expressway on the east in Sunnyvale, California. The site is currently occupied by industrial and office buildings with associated parking lots and landscaped areas. The site is located within a mixed-use area and surrounded by office buildings and residential properties. Based on U.S. Geological Survey (USGS) topographic maps, site grades varied from approximately Elevations 32 to 50 feet. In general, the site appeared relatively level. The approximately 18 feet of topographic relief slopes down from the southwest to northeast corners of the property.

2.3 Subsurface

2.3.1 General

Based on our previous explorations in the site area and alluvium thickness maps of Santa Clara County (Rogers and Williams, 1974), the site is underlain by fluvial and inter-fluvial deposits, which consist of fine-grained sand, silt, clay, organic clay, and silty clay, extending to depths in excess of 500 feet. In general, the silt and clay deposits are anticipated to be medium stiff to very stiff, and the sand deposits are anticipated to be medium dense to dense.

2.3.2 Anticipated Subsurface Conditions

Our review of the subsurface information in our files from the nine sites located within ½-mile of the site, indicates the site is likely blanketed by stiff to hard, moderate to high plasticity clays to a depth between about 2 to 9½ feet. Below this clay layer, we anticipate that interbedded layers of medium stiff to very stiff, low to moderate plasticity clays, and medium dense to very dense sands would be encountered to a depth of 120 feet, the maximum depth of our previous exploration in the site area.

Plasticity Index (PI) tests performed on clayey soil samples during our previous investigations in the site area generally exhibited a range of PIs from 22 to 48 for the near-surface clayey soils, which indicates the near-surface soils have moderate to high plasticity and expansion potential. The test results for the relatively deeper clayey soils, between depths of 7½ to 46 feet, exhibited PIs between 5 and 31, indicating the subsurface clayey soils are predominantly low to moderately expansive.

Since portions of the site were developed, we anticipate that undocumented fills, debris, and abandoned underground utilities would likely be present. Other undocumented backfills likely exist in the project area due to construction of depressed loading docks, UST's and underground utilities. Our previous investigations performed within the developed portions of the site encountered shallow fill to a depth up to 2 feet. One of our previous borings was performed at a former gas station, located on the southeast corner of East Duane Avenue and DeGuigne Drive, which encountered undocumented fill to a depth of about 13 feet within a former Underground Storage Tank (UST) excavation area.

2.4 Ground Water

During our previous investigations in the site vicinity, free ground water was encountered at depths ranging from approximately 7 to 14½ feet below the existing ground surface. According to Plate 1.2 of the Seismic Hazard Zone Report 060, prepared by the California Geological Survey (CGS, 2003), historic high ground water level in the site vicinity is considered to be on the order of 9 to 11 feet. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, regional influence, and other factors not evident at the time the ground water level measurements were made.

2.5 Site Infiltration

As the site is likely blanketed by moderate to high plasticity clays, we judge the site infiltration rate will be very low for any proposed site detention/retention facilities. As discussed above, free ground water was encountered at depths between 7 to 14½ feet in the site vicinity, and the historic high ground water level in the site vicinity is considered to be on the order of 9 to 11 feet. The Regional Water Quality Control Board (RWQCB) requires that a minimum of 10 feet be maintained between the seasonal high ground water level and the bottom of any infiltration facility. Therefore, pre-treatment of pavement runoff water and potentially roof runoff prior to entering any infiltration facilities would likely be required.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 2. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) nor is it located in a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). As shown on Figure 2, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to all developments in the San Francisco Bay Area. The "Seismicity" section that follows summarizes potential levels of ground shaking at the site.

3.3 Liquefaction

3.3.1 General Background

The site is located within an area zoned by the State of California as having potential for seismically-induced liquefaction hazards (CGS, 2003 – Mountain View Quadrangle) and in a Santa Clara County liquefaction hazard zone (SCC, 2002). During cyclic ground shaking, such as during earthquakes, cyclically induced stresses can cause increased pore water pressures within a soil matrix, resulting in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure under moderate to high shear stresses (Youd et al., 2001). Liquefied soil can also settle (compact) as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

3.3.2 General Subsurface Conditions

As noted in the subsurface description above, our previous investigations at portions of the site and in the site vicinity encountered interbedded layers of sands and clays beneath the surficial fills and near-surface clays. The sand layers generally ranged from a few inches to 19½-feet-thick. In general, the sand layers were medium dense to very dense, and the clay layers were medium stiff to very stiff.

3.3.3 Anticipated Results

There is a moderate to high risk that liquefaction will occur during strong seismic shaking. If the magnitude of the estimate total and differential post liquefaction settlement exceed what shallow foundation designs deem tolerable for combined static and seismic differential settlements, ground improvement or deep foundations may be required. Our previous analyses indicate that liquefaction-induced settlements for the site area are generally on the order of 1 inch or less with differential movement of about ½-inch between independent foundation elements or over a horizontal distance of 50 feet. Therefore, we anticipate that the potential for liquefaction-induced settlements at the project site would be similar to the results above; however, the possibility of higher seismic settlements at some portions of the project site should not be ruled out. The estimated liquefaction-induced settlements should be confirmed by additional explorations during design-level geotechnical investigations.

The methods of analysis used to estimate total settlement do not take into account the possibility of surface ground rupture. In order for liquefaction-induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer. Based on work by Youd and Garris (1995), a capping layer of non-liquefiable material on the order of 4½ to 5 feet thick is adequate to prevent the occurrence of ground surface rupture for a liquefiable layer on the order of 2 to 3 feet in thickness. Our previous explorations in the site vicinity indicate there is enough of non-liquefiable material capping the potentially liquefiable sand strata; however, relatively shallow sand layers were encountered that will need to be evaluated for liquefaction during design-level investigations. We also reviewed the Historical Ground Failures in Northern California Triggered by Earthquakes by Youd and Hoose (1978), which mapped recorded occurrences of liquefaction, ground rupture and lateral spreading. The maps do not indicate ground rupture or lateral spreading occurred during the 1906 San Andreas Earthquake. So while the potential for ground rupture should be evaluated, at this time the risk of occurrence appears to be low.

3.4 Differential Compaction

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. Loose sands could also undergo minor compaction due to seismic shaking. Provided any undocumented fills are removed and replaced as engineered fill and shallow loose sands, if encountered, are mitigated, we judge the potential for differential seismic compaction at the site to be low.

3.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. Since there are no creeks or open bodies of water within an appropriate distance from the site, we judge the probability of lateral spreading occurring at the site during a seismic event to be low.

4.0 SEISMICITY

4.1 Regional Active Faults

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The San Andreas Fault, which generated the great San Francisco earthquake of 1906, passes about 15 kilometers southwest of the southern site boundary. Other major active faults in the area are the Hayward Fault, located about 16 kilometers northeast of the northern site boundary, and the Calaveras Fault, located about 18 kilometers northeast. A potentially active fault closest to the site is the Monte Vista – Shannon Fault, located about 9½ kilometers to the southwest.

4.2 Maximum Estimated Ground Shaking

The Probabilistic Seismic Hazard Analysis (PSHA) performed by the CGS estimates a pseudo-peak horizontal acceleration of about 0.52g, with a 10 percent chance of exceedance in 50 years, for the site area. Pseudo-peak ground accelerations have been normalized to a 7.5Mw seismic event, weighted to account for regional seismic activity and fault distances.

4.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003), referred to as WG02, estimates there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2002 and 2031. This result is an important outcome of WG02's work, because any major earthquake can cause damage throughout the region.

This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

4.4 California Building Code (CBC) Site Seismic Coefficients

The CGS has issued maps locating "Active Fault Near-Source Zones" to be used with the 2001 CBC ("Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," CDMG/ICBO February 1998). Faults are classified as either "A," "B," or "C" as shown below. Only faults classified as "A" or "B" are mapped since faults classified as "C" do not increase the near-source factor.

Table 1. Seismic Source Definitions

Seismic Source Type	Seismic Source Description	Seismic Source Definition*	
		Maximum Moment Magnitude, M	**Slip Rate, SR (mm/yr)**
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	M ≥ 7.0	SR ≥ 5
B	All faults other than Types A and C.	M ≥ 7.0 M < 7.0 M ≥ 6.5	SR < 5 SR > 2 SR < 2
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	M < 6.5	SR ≤ 2

*Note: Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining seismic source type.

The following table lists Type A and Type B faults within 25 kilometers of the site:

Table 2. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (kilometers)
**Monte Vista - Shannon	B	9.4 - 10.8
Hayward (Southeast Extension)	B	12.3 - 13.8
*San Andreas (1906)	A	14.6 - 16.2
Hayward (Total Length)	A	16.0 - 17.5
Calaveras	B	17.7 - 19.3

*Nearest Type A fault
**Nearest Type B fault

The CBC describes the procedure for determining soil profile types S_A through S_F in accordance with Section 1636.2 and Table 16-J. Based on our previous explorations and alluvium thickness maps of Santa Clara County (Rogers and Williams, 1974), the site can be characterized as soil profile type S_D generally described as a stiff soil profile. Based on this information and local seismic sources, the site may be characterized for design based on Chapter 16 of the 2001 CBC using the information in Table 3 below.

Table 3. 2001 CBC Site Categorization and Site Seismic Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_D
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Monte Vista - Shannon
Seismic Source Type (Table 16-U)	B
Distance to Seismic Source (kilometers)	9.4 - 10.8
Near Source Factor N_a (Table 16-S)	1.00
Near Source Factor N_v (Table 16-T)	1.00 - 1.02
Seismic Coefficient C_a (Table 16-Q)	0.44
Seismic Coefficient C_v (Table 16-R)	0.64 - 0.66

5.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

5.1 General

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the East Sunnyvale ITR development. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. We recommend that final geotechnical investigations be performed once the site development plans have been finalized for each future project area. Results from the final investigations will be used to confirm preliminary findings and to develop detailed geotechnical recommendations for final design.

The primary geotechnical concerns at the site are as follows:

- Potentially liquefiable soils
- Undocumented fill and former UST backfill
- Expansive soils
- Shallow ground water

A brief description of each potential issue is presented below. As previously discussed, we are concurrently performing an Environmental Hazardous Material Evaluation for the site. The environmental concerns are presented in our environmental report under a separate cover.

5.1.1 Potentially Liquefiable Soils

As previously discussed, our analyses during previous investigations in the site vicinity indicate that some of the sand layers may theoretically liquefy and result in some post-seismic total and differential settlements. Therefore, foundations should be designed to resist or accommodate this movement. As part of the liquefaction evaluation for each future project area, any shallow sand layers should also evaluate for liquefaction-induced ground rupture.

5.1.2 Undocumented Fill and Former UST backfill

Undocumented fill is likely present at developed portions of the project site. Our previous explorations at the former gas station, located within the project area encountered shallow fill to a depth of about 2 feet and loose backfill within the former UST excavation area to a depth of about 13 feet. Undocumented fill may impact surface improvements such as sidewalks and at-grade pavement areas, as well as structure foundation areas. All fills should be removed within building areas. If desired to reduce the risk of uneven settlement of at-grade improvements such as pavements and sidewalks, we recommend that fills outside the proposed building footprints be removed and replaced as engineered fill. Dewatering to remove the backfill material in the former UST excavation area may be necessary.

5.1.3 Expansive Soils

To reduce the potential for damage to the planned structures due to the presence of moderately to highly expansive surficial soils, we recommend slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill (NEF) and that footings extend below the zone of seasonal moisture fluctuation. As an alternative to structures with footings and slabs-on-grade over NEF, post-tensioned mat foundations may be desired.

5.1.4 Shallow Ground Water

If the proposed buildings are constructed with a fully or partially below-grade basement or garage, shallow foundations will bear at or below the ground water table. Depending on the building loads and finished foundation elevations, the foundation settlement under static loads may be significant enough that grid footings, conventionally reinforced mat foundations, or deep foundations may be necessary. Details settlement analyses considering the range of anticipated loading should be performed during design-level investigation.

Ground water may also significantly impact grading and below-grade construction. These impacts typically consist of potentially wet and unstable subgrade soils, difficulty achieving compaction, and difficult underground utility installation. As previously discussed, ground water was encountered in our explorations at a depth as shallow as about 7 feet below the existing ground surface and ground water levels may fluctuate seasonally. Therefore, the contractor should be aware that excavations extending near or below ground water may need to be stabilized and/or dewatered to facilitate placement and compaction of structures and fill. Contractors should anticipate difficulties reworking the UST backfill material excavated below the ground water table at the former gas station.

Depending on the embedment depth of planned basements or below-grade garages, inclusion of water-proofing and design for hydrostatic uplift and wall pressures in the project plans may be needed.

5.2 Design-Level Geotechnical Investigation

This feasibility report was prepared using limited previous on-site and nearby subsurface data. All future projects within the Sunnyvale ITR site area should have design-level geotechnical investigations performed. The geotechnical scopes of work should be based on site development plans and addressing the geotechnical concerns raised in this report. The geotechnical engineer should also be retained to review the final construction plan and specifications, and perform observation and testing during construction.

5.3 Earthwork

No unusual earthwork requirements are anticipated. Backfilling of holes or pits resulting from demolition, removal of existing undocumented fills, and removal of existing buried utilities should be carried out under the geotechnical engineer's observation, and all of the backfill should be properly compacted and tested during placement.

Surficial native clayey are moderately to highly plastic and are difficult to compact and maintain a stable subgrade when the soil is several percent above the laboratory optimum moisture content. In addition, since the surficial native soils have a relatively high moisture content in winter months and due to regular irrigation, earthwork contractors should anticipate that these soils may require drying (aeration) prior to use as engineered fill or subgrade preparation even during summer months. Consideration should be given to the use of light weight grading equipment. The use of heavy vibratory equipment will tend to de-stabilize clays with high in-situ moisture contents.

Moderately to highly plastic soils are also susceptible to volumetric change during wetting and drying. Foundation excavations and slab-on-grade areas should be kept moist prior to placing concrete. If the soil is allowed to dry significantly, re-moisture conditioning can require days of effort.

Difficulties may also be experienced where below-grade excavations extend near or below ground water. Stabilization and local or areal dewatering could be required. Stabilization techniques may include sub-excavation and replacement with about 12 to 18 inches of crushed rock over stabilization fabric, or if the area is larger enough, chemical treatment. The decision of which option is the most desirable and effective should be made on a case by case basis by the geotechnical engineer.

5.4 Building Foundations

In our opinion, the proposed buildings may be supported on conventional spread footing foundations provided that estimated total and differential settlements due to static loads and liquefaction are tolerable. Detailed settlement analyses should be performed for each project once building loads and foundation elevations are established. As discussed above, foundation settlement may indicate the need for grid footings, mat foundations or deep foundations if total and differential static and seismic settlements are not tolerable to isolated footings.

5.4.1 Footings

Since the surface soils at the site are likely to be moderately to highly expansive, the bottoms of the footings are anticipated to extend at least 24 inches below the lowest adjacent finished grade, considered as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape top soil, whichever is lower. These relatively deeper footings are recommended to place bearing surfaces below the zone of significant moisture fluctuation to reduce the effects of heave and shrinkage.

We anticipate that footings would be capable of supporting maximum allowable bearing pressures of 2,500 pounds per square foot (psf) for dead load, 3,750 psf for combined dead and live loads, and 5,000 for all loads including wind or seismic. Detailed bearing capacity analyses in conjunction with settlement analyses should be performed during design level investigations to refine these values as appropriate.

5.4.2 Reinforced or Post-Tensioned Mats

Alternatively, the structures may be supported on conventional reinforced mat or post-tensioned mat foundations. We anticipated that reinforced or post-tensioned mats would be capable of supporting an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized bearing pressures of up to 2,500 psf at column or wall loads.

Due to the moderate to high expansion potential of surficial soils, we recommend that finished pads be moisture conditioned to at least 3 percent over optimum in the upper 12 inches of the building pads prior to placing the moisture barrier system. The moisture content of the finished pads should be checked within 24 hours prior to the construction of the moisture barrier.

5.4.3 Deep Foundations

If significant building loads are anticipated and/or the total and differential settlements due to static loading and liquefaction are not tolerable from a structural viewpoint, deep foundations may be required to support the proposed buildings. Design-level geotechnical investigations should recommend a suitable deep foundation system, if necessary, once final development plans and building loads are available for review.

5.4.4 Lateral Loads

We anticipate that lateral loads may be resisted by friction between the shallow foundations and the supporting subgrade. On a preliminary basis, a maximum allowable frictional resistance of about 0.25 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We anticipate that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) may be used in design. The upper 12 inches should be neglected when calculating lateral resistance unless covered by concrete slabs or pavements.

5.4.5 Interior Slab-on-Grade Floors

Since the expansion of the surficial clayey soils may vary across the site, we anticipate that interior concrete slab-on-grade floors used in conjunction with shallow footings will likely need to be supported on at least 18 to 24 inches of non-expansive fill (NEF) to reduce the likelihood of slab damage from heave. If desired to limit floor wetness in habitable areas, a slab moisture protection system consisting of a vapor retarder meeting at least ASTM E1745 Class C requirements and a maximum permeance of 0.03 perms underlain by 4 inches of crushed rock can be used. The crushed rock may be included as part of the NEF requirements.

5.4.6 Exterior Flatwork and Sidewalks

Due to the moderate to high expansion potential of surficial soils at the site, we anticipate that exterior flatwork and sidewalks will likely be required to be supported on at least 6 to 12 inches of Class 2 aggregate base compacted to at least 90 percent relative compaction.

5.5 Surface Improvements

We anticipate that new pavements will be designed in accordance with City of Sunnyvale standards. Based on the soils encountered in our previous investigations in the site vicinity and our engineering experience, we judge an R-value of 5 to be appropriate for preliminary design of asphalt and concrete pavements.

6.0 LIMITATIONS

This report has been prepared for the sole use of David J. Powers & Associates, specifically for preliminary planning of the East Sunnyvale ITR Project to be included in the EIR for future development at the project site in Sunnyvale, California. The opinions presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, preliminary conclusions and recommendations contained in this report are based upon the information obtained from our previous investigation on-site and nearby, which includes data from widely separated locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between explorations do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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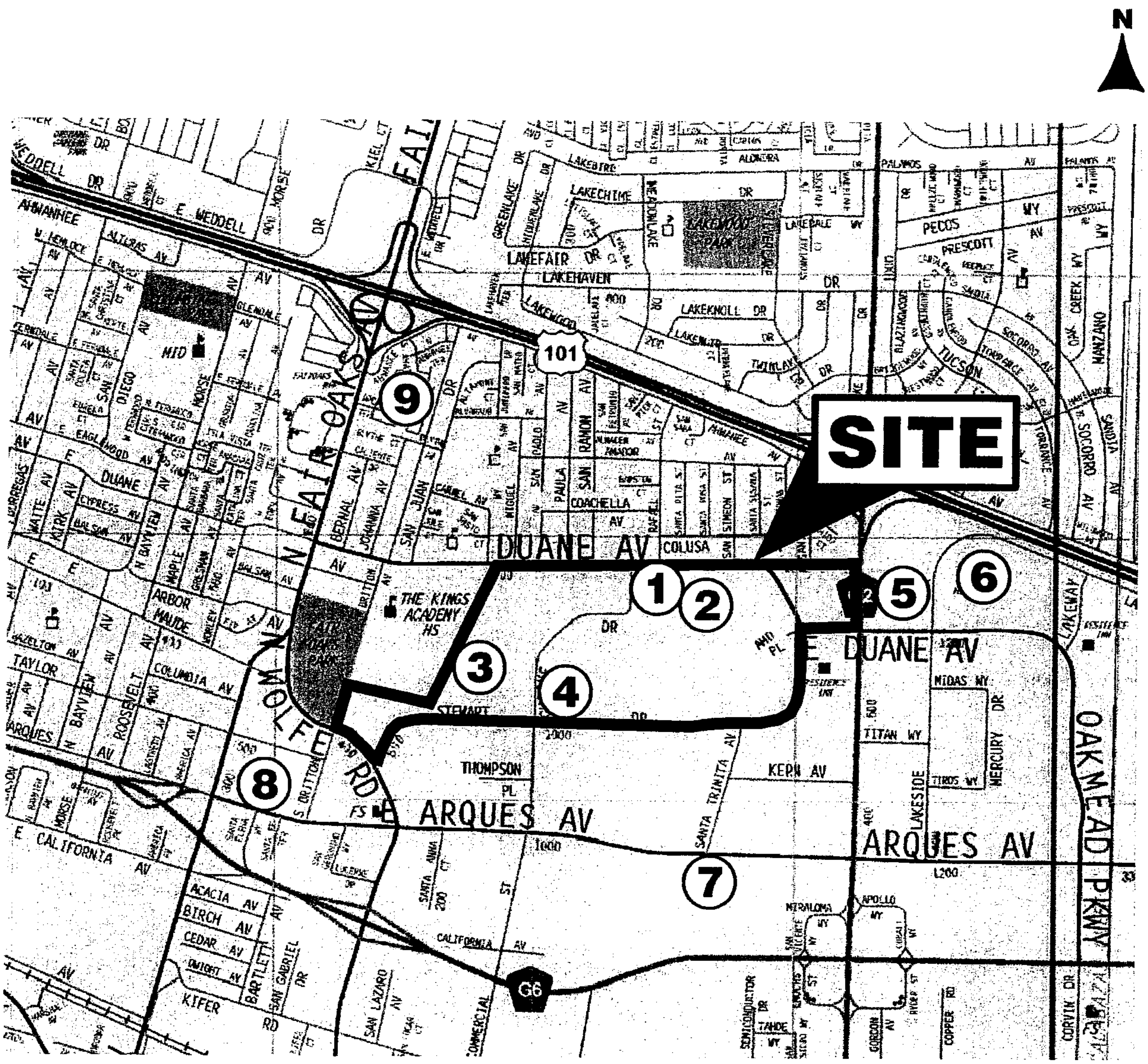

LEGEND

1. 920 AND 948 E. Duane Avenue
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5. Oakmead Apartments
6. Crescent Residential Development
7. Sunnyvale Research Center Building
8. Classics at Arques Residential Development
9. Compass Place Condominiums

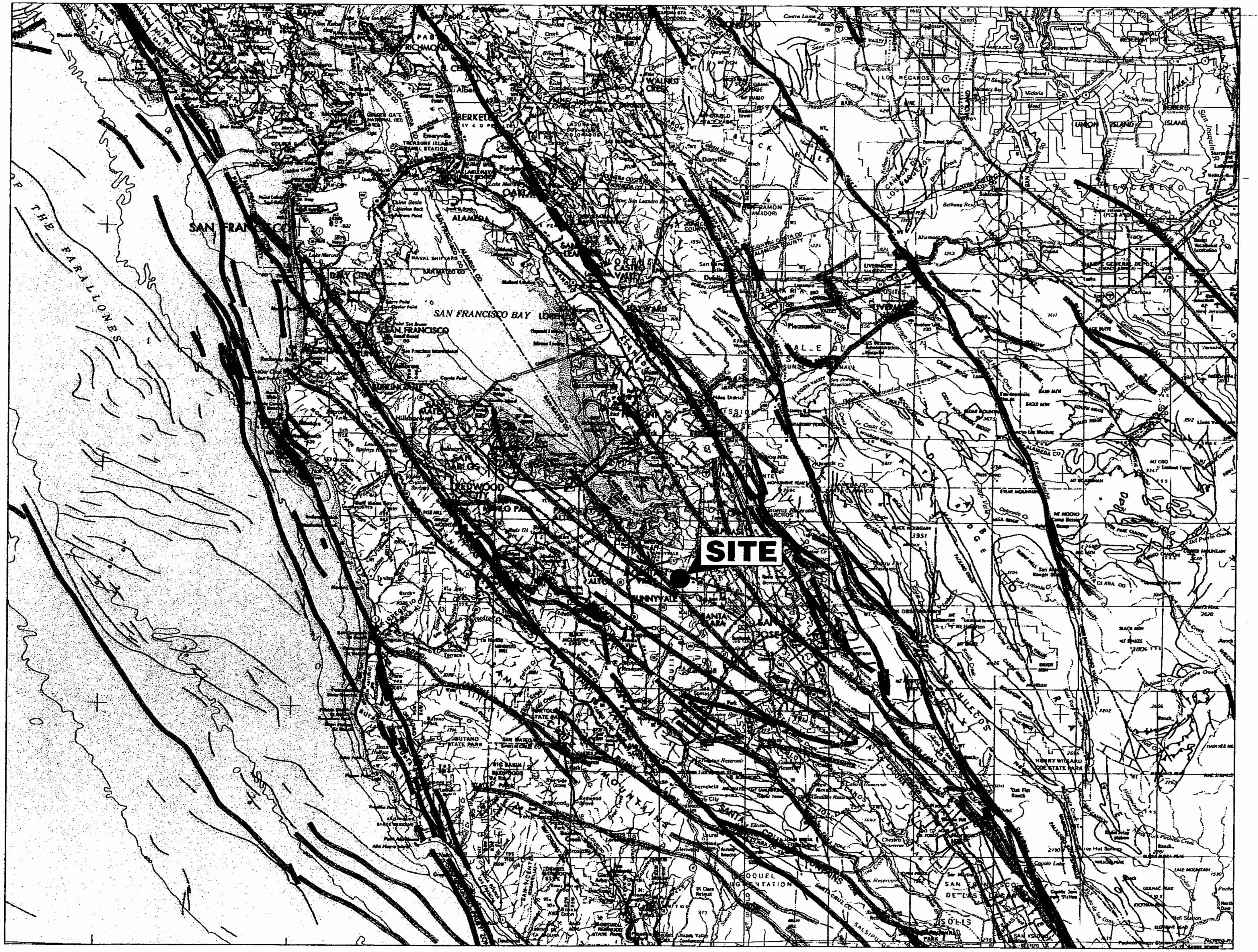
1/06*EB

VICINITY MAP

EAST SUNNYVALE ITR PROJECT
Sunnyvale, California

TRC Lowney

FIGURE 1
858-47A



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000			Displacement during Holocene time.[2]
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.[3,4]
	Early Quaternary[5]		2,000,000			Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		5,000,000			
	Miocene					Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122–A1–TF–250–00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120–A1–TF–250–00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000–meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

REGIONAL FAULT MAP

EAST SUNNYVALE ITR PROJECT

Sunnyvale, California

TRC Lowney

FIGURE 2

858-47A

Division of Mines and Geology, James F. Davis, State Geologist

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